CHAPTER 7

EVALUATION PROCESS

7-01. Nature of Dam Safety Problem.

a. Hydrologic Deficiency.

The hydrologic and hydraulic studies associated with this report involved the application of current criteria and guidance to determine the inflow design hydrograph, determine the required freeboard for the dam, evaluate the capacity of the emergency spillway, evaluate the performance of that spillway in the case of unsatisfactory gate performance, evaluating dam performance under dambreak conditions, estimating downstream flows and stages under various conditions with and without a dambreak, and mapping the downstream floodplains.

(1) Inflow Design Hydrograph (IDH). Guidance for this effort was taken from ER 1110-8-2(FR) *Inflow Design Floods for Dams and Reservoirs.* A basin runoff model of the Big Blue River basin was constructed by the staff of the District using computer program HEC-1. The maximum probable rainfall for that model was established using the data and techniques incorporated in *Hydrometeorological Report No. 51 – Probable Maximum Precipitation Estimates, United States East of the 105th Median* and computer program HMR 52. Twelve centerings of the storm over the basin were made to determine the maximum inflow hydrograph into Tuttle Creek Lake. After the maximum inflow hydrograph was determined, the peak discharge for the event was increased by 25% while the volume of the hydrograph was maintained. This was done according to guidelines, and is a compensation for the fact that great flood waves move through the basin more rapidly than normal floods, due to the greater depth involved. Flood hydrographs for floods less than the full PMP were determined by multiplying the ordinates of the full PMP hydrograph by factors of 0.9, 0.8, 0.7, etc.

(2) Wind-Wave Analysis. The required freeboard for dams is defined as the difference between the maximum static water surface in the lake and the top of the dam. The procedures used to estimate the required freeboard have changed since the project was designed. Current guidance for these determinations is given in the U. S. Army Corps of Engineers *Shore Protection Manual*, EM 1110-2-1414 *Water Levels and Wave Heights for Coastal Engineering Design*, EM 1110-2-1420 *Hydrologic Engineering Requirements for Reservoirs*, and EM 1110-2-2904 *Design of Breakwaters and Jetties*. Using this guidance, the fetch length was determined to be 5.5 miles, the design wind velocity was found to be 35 mph for a duration of 1 hour-35 minutes, and the overall wave runup was found to be 4.6 feet.

(3) Capacity of the Emergency Spillway. This study used some of the newly available mixed flow capabilities of computer program HEC-RAS to analyze sub critical flow down the exit chute, mixed flow through the 18 bay tainter gate structure, super critical flow in the approach channel, and the stage and energy losses necessary to accelerate quiescent lake water into the upstream approach channel. The discharge versus water surface elevation for the approach channel immediately upstream of the tainter gate structure was found to be quite similar to the relationship currently in use at the project but, when the water surface drawdown

necessary to introduce water into the upper end of the approach channel was taken into account, the overall spillway rating was altered. The net effect of these changes was to raise the required lake water surface elevations for large spillway discharges.

(4) Unsatisfactory Gate Performance. The operating procedure for Tuttle Creek Dam for severe lake inflow events calls for allowing the lake surface to raise to the elevation of the top of the tainter gates while those gates are resting on their sills. As inflow increases, the tainter gates are raised until they finally clear the nape and are placed in the fully raised position. This procedure exposes the tainter gates to the full static load before they are moved. It is imperative that these gates operate when called upon, so tainter gate reliability is a critical element in the overall safety of Tuttle Creek Dam. It is noted that there are some legitimate reservations about the overall reliability of these gates (see Section 3-03.c.). In order to evaluate the hydrologic consequences of impaired tainter gate performance, the HEC-RAS model described above was operated with the assumption that two of the 18 tainter gates failed. It is noted that, with all gates operable, the full IDH for the full probable maximum flood infringes on the required dam freeboard, but the static water surface does not raise above the top of the dam. With two gates disabled, the static water surface for the same flood raises above the top of the dam.

(5) Dam Break Modeling. The District contracted with HDR Engineering, Inc. for a study which included: (a) developing a dambreak model for Tuttle Creek Dam, (b) developing a unsteady flow model of the downstream floodplains (Kansas and Big Blue Rivers), (c) mapping of the downstream floodplains with a delineation of the flooded areas, and (d) an analysis suitable to support the base safety condition for the project. The consultant's report is included in Appendix III. The status of this study and the overall H&H work on this project was reviewed in an In-progress Technical Review conducted on 26 July 2001 in the District office. Due to the obvious increase in the flooded area footprint associated with a dambreak event during the full PMP in the City of Manhattan KS, the use of the full PMP as the base safety condition was approved without conducting a full population-at-risk and probable fatalities study.

(6) Downstream Flows. Discharges, hydrographs and profiles for floods representing the full probable maximum flood, and floods representing 90%, 80% and 70% of the full probable maximum flood are presented in the HDR report referenced in the preceding paragraph. Data for each flood is presented assuming the dam remains intact, and assuming the dam is breached.

(7) Mapping of Floodplains. Mapping of the flood plains of the Big Blue River and Kansas River downstream of Tuttle Creek Lake are included in the HDR report in Appendix III.

(8) Revised Inflow Design Hydrograph (IDH). The revised IDH, which incorporated the wind-wave analysis and spillway capacity analysis referred to above, was approved by the Northwestern Division by letter of 18 September 2001.

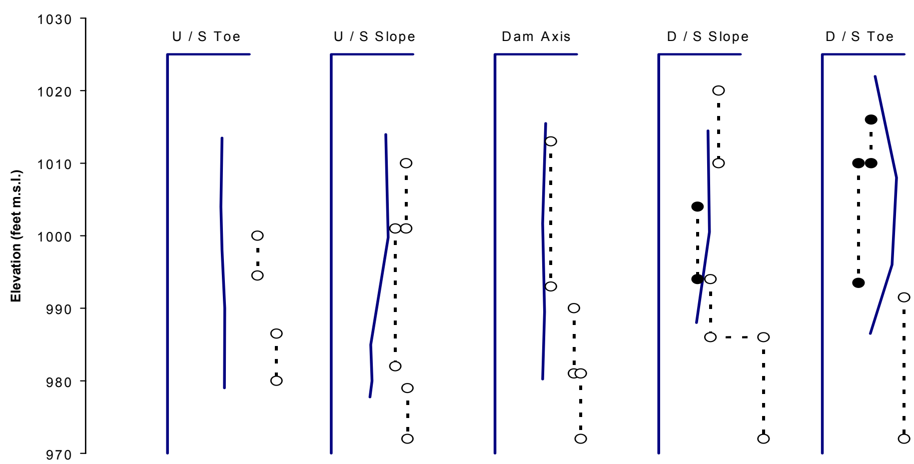
b. <u>Seismic Deficiency</u>.

(1) Extensive investigation using Standard Penetration Tests (SPT), Cone Penetrometer Tests (CPT), Shear Wave Velocity measurements, and dynamic laboratory tests demonstrated that the foundation soil in the vicinity of both upstream and downstream toes of the dam and as far under the dam as the mid-slope is liquefiable under the action of the design earthquake.

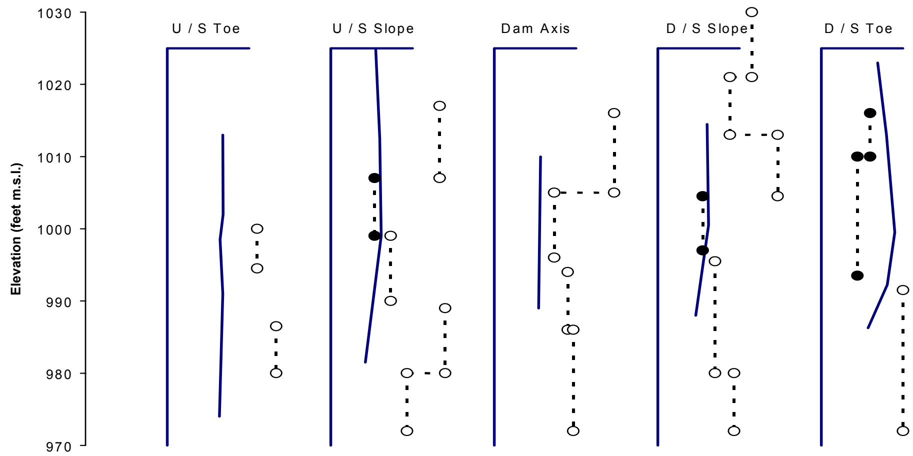
(2) The results of the analysis based on SPT data are summarized in Figure 7.1. The threshold values of "$(N_1)_{60}$ adjusted for fines", were represented in the figure by solid lines. Actual values of "$(N_1)_{60}$ adjusted for fines" were calculated as average values for relatively uniform layers. They were plotted in the figure with circles connected by dashed lines. Layers represented by circles located to the left of the solid lines have the factor of safety less than 1.1 against liquefaction and are potentially liquefiable.

(3) Based on Figure 7.1.c, it is evident that between stations 35+00 and 70+00 sand in the foundation is liquefiable between elevations 1010 and 994 under the lower portion of the upstream slope and between elevations 1010 and 990 under the lower portion of the downstream slope.

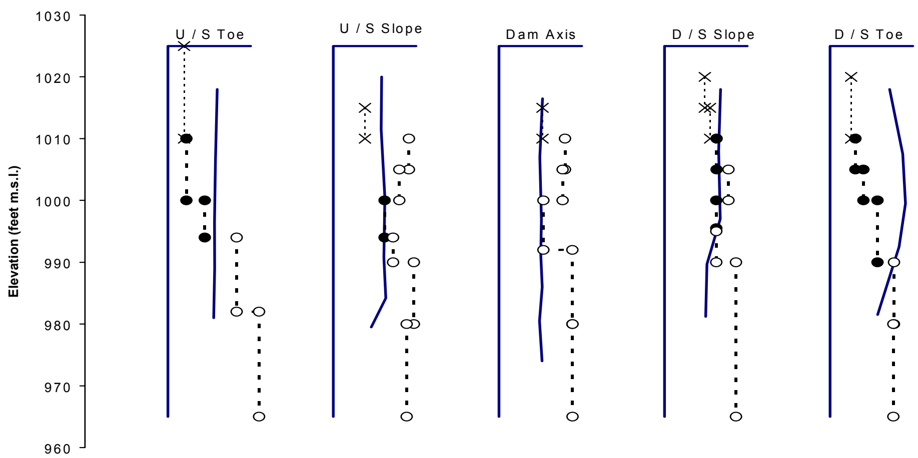
(4) Figure 7.1.a&b shows that in the zone of closure section (stations 33+00 to 35+00) and to the west (stations 25+00 to 33+00) the sand under the downstream toe and slope is liquefiable and on the upstream side is non-liquefiable or marginally liquefiable. However, in these reaches the soil liquefiability was evaluated based on only one boring in each characteristic location (toe, mid-slope) so that these results should be verified by additional investigation in Phase III, Detailed Design and Plans and Specifications.



a. Stations 25+00 to 33+00.



b. Stations 33+00 to 35+00.



c. Stations 35+00 to 70+00.

Fig. 7.1. Comparison between threshold "$(N_1)_{60}$ adjusted for fines" values and actual data for station ranges. ●…●: liquefaction probable. ○…○: liquefaction not expected. x…x: Cohesive (clayey) soils.

c. <u>Other Unsafe Conditions; Tainter Gates</u>.

The design of the Tainter gates did not consider wave loading. Other loads used in the original design do not meet current criteria. These loads will impart bending moments on the struts in the weak axis, potentially resulting in an overstress condition. Overstressing of the gates could result in failure of one or more gate members. This situation could result in uncontrolled releases through the gate bays or the inability to make controlled releases through the gates. This situation has occurred at a Bureau of Reclamation Dam and numerous Tainter gates at Corps of Engineers projects have been, or are being, modified and strengthened to address this condition.

7-02. Extent of Deviation from Current Criteria.

a. Hydrologic Adequacy.

Pertinent guidelines for the safe operation of large earth dams require that the crest of the dam be at or above the maximum water surface in the lake with a provision for freeboard based on wind driven waves. In the case of Tuttle Creek Dam, the crest elevation of the dam is 1159, and the freeboard required to contain wind/wave action is 4.6 feet. This means that the static lake water surface elevation should not exceed elevation 1154.4. For the inflow design hydrograph as discussed in section 7-01.a., the static water surface will rise to elevation 1156.8, which does not meet this requirement by 2.4 feet. This elevation is based on the assumption that all 18 tainter gates are fully functional. Should two of the spillway tainter gates malfunction during the IDH event, the static lake water surface will rise to elevation 1159.1. Malfunctioning of more than two gates will simply drive more water over the crest of the dam.

b. Seismic Stability.

The consequence of extensive liquefaction of the foundation soil (see paragraph 7-01.b and Figure 7.1) is a high potential for large deformations and complete failure of the dam. As shown before in this report (Chapter 3, Figure 3.1) the factor of safety for post-earthquake limit equilibrium is on the order of 0.6 to 0.8 for at least the reach between stations 35+00 and 70+00.

c. Other Features; Tainter Gates.

The Tainter gate design did not include wave loading. Other original load cases do not meet current criteria. Although trunnion friction was used in the original design the magnitude and application of the friction does not meet current design criteria.

U.S. Army Corps of Engineers Engineering Manual 1110-2-2702, "Design of Spillway Tainter Gates" Paragraph 3-4.b.(1) (f), "Trunnion pin friction loads F_t" defines the current criteria for design and evaluation of Tainter gates. These criteria require the consideration of friction loads at the trunnion pin in the structural design of the gate strut bracing. A coefficient of friction of 0.3 is the upper bound design criteria for trunnion pin bushing material that may be slightly worn or improperly maintained. Lower design coefficients of friction are allowable where lubricated bronze or aluminum bronze bushings are present.

The 18 Tainter gates at Tuttle Creek Dam were constructed with lubricated bronze bushings and are of an age that they should be considered "slightly worn". Additionally, the Operation and Maintenance Manual only required minimum maintenance of the trunnion bearings. It is therefore considered appropriate to apply the required design coefficient of friction of 0.3 to the Tuttle Creek Tainter gates. A memorandum from CEMRD-ED-TS to CEMRK-ED on 05 August 1994 specifically requested that the Tainter gates at Tuttle Creek Dam be evaluated considering trunnion friction loads.

The spillway gates were not damaged during lifting with the pool near the top of the gates in 1993. However, considering a trunnion bearing friction coefficient of friction of 0.3, a preliminary 3-D STAAD model of the Tainter gates indicates that the lower strut arms would be overstressed with a Load Resistance Factor Design (LRFD) interaction value of 1.24 and the bottom horizontal girder would be overstressed with an LRFD interaction value of 1.18 if the gates are lifted with water at the top of the gates. The failure of either of the overstressed members would lead to the collapse of the gate structure.

Monitoring of friction loads and strain in the gate arms has not been performed to evaluate the current friction loads on the gates. Monitoring of trunnion friction loads would only be applicable and accurate if pool loading were present on the gate. Since the Tuttle Creek Spillway is a controlled discharge spillway and water has only been against the Tainter gates twice in the approximate 40-year history of the project (and only once to near the top of the gates), the conditions that would allow this monitoring to be performed are extremely rare.

Loading conditions for Tainter gate evaluation will meet current criteria. A revised wind/wave analysis using current design criteria is being performed.

Earthquake design loads are also required to be considered by Engineering Manual 1110-2-2702, "Design of Spillway Tainter Gates" for the operational basis earthquake (OBE). However, since the Tuttle Creek spillway is a controlled discharge spillway used only during extreme flood events, it is considered unlikely that an earthquake will occur concurrent with an extreme flood event. The spillway gates are considered "non-critical" structures and are not designed to withstand earthquake forces.

The hydraulic analyses of the spillway indicated that the PMF flow would pass within 2 feet of the bottom of the gate at the center of the gate bay. There was concern that mounding at the pier nose and pier influences on the flow could cause the flow to impact the corner of the gates. In December of 2001, a gate exercise was conducted to determine if the gates could be lifted above the design elevation of 1140. It was determined that the gates can be lifted to 1141.4, thus providing approximately 3.4 feet of clearance between the bottom of the gate and flow through the gate bay. This clearance is considered adequate to address the uncertainties associated with pier effects.

During the December 2001 gate exercise it was determined that the gate hoists brakes were sufficient to support the gates during an extreme spillway discharge A new dogging system for this purpose is not necessary.

7-03. <u>Nature of Damages Associated with Dam Failure</u>.

a. <u>Potential Economic Losses</u>.

With dam failure all current annual project benefits would be lost including flood control, recreation, navigation, water supply, water quality, and fish and wildlife. In addition to the annual benefits lost there would be one-time failure specific flood damages that would occur due to the extensive flooding resulting from the outflow from the dam breach.

A dam breach would flood the residential and commercial areas downstream, and would have many other adverse social and economic consequences. Dam failure would result in direct loss of life in the residential, commercial and industrial areas, potential disruption of services, critical facilities and access thereto, extensive property losses, and extensive environmental losses with high costs for mitigation.

Structures and contents in the flooded area would be extensively damaged or destroyed. Public property such as roads and bridges would be damaged, potentially interrupting vehicular traffic. Recreation related businesses located above and below the dam would be severely disrupted. Residents' lives would be disrupted, with potentially traumatic and negative emotional experiences of personal loss and of dealing with flood clean-up requirements. Public health and relief needs of affected people would be another adverse impact.

Upstream of the dam, Tuttle Creek Lake offers a variety of water-based recreation opportunities that are an important part of the local economy. Visitation for FY 2000 exceeded 2,654,000 visitor hours. Four of the park areas in the project are operated as State Parks by the Kansas Department of Wildlife and Parks.

The area downstream of Tuttle Creek Dam was divided into 25 socioeconomic reaches for purposes of this analysis. Inundation maps (see Attachment A) were prepared for a 500-foot seismic breach width that develops to the base of the reservoir in five hours. Economic damage and loss of life reaches maps are included in Attachment B. A significant portion of the downtown business district and commercial area of the City of Manhattan, Kansas is located in the inundated area. Portions of Topeka, Kansas, Lawrence, Kansas and other smaller communities along the Kansas River would also be impacted. Commercial and residential losses are based in part on the total investment value of structure and contents. Residential investment values used were taken from census tract data, and a percentage of structure value was used for estimating content value. For commercial development, an average value of commercial investment per acre was applied to the flooded acres in the commercial/industrial land use category. This value was determined based on previous economic studies in the area and a recent limited field survey of the area. Agricultural losses are based on damage to crops, although damage to livestock, lands, machinery, buildings and fences could be incurred. The number of acres affected was estimated by measuring the cropland and grassland acres within the footprint of the flooded area. The crop mix used to determine the lost production value was based information from previous studies about the cropping patterns in the area. Table 7-1 displays by reach the inventory of land usage, investment, and crop production values in the area that would be inundated with a 500-foot seismic breach in the dam.

Table 7-1. Economic Inventory Downstream of Tuttle Creek Dam.

Seismic Breach Inundated Area (500-foot breach width to base of reservoir in 5 hours)

Reach	Reach Lower Boundary (R.M.)	Comm/Ind Develop-ment (Acres)	Com/Ind Value - Struc & Contents ($1000)	Cropland/ Grassland (Acres)	Crop Production Value ($1000)	Residential Develop-ment (Acres)	Residential Value - Struc & Contents ($1000)	Number of Housing Units
1	BBR 5.3 (Manhattan-part)	71	$71,000	2,429	$318.8	17	$37,936.0	582
2	BBR Mouth/ KSR 147.3 (Manhattan-part)	311	$311,000	2,252	$318.2	291	$90,888.9	1,705
3	KSR 148.5 (Manhattan-part)	354	$354,000	490	$63.9	428	$20,157.6	2,892
4	KSR 155.2	0	$0	4	$0.4	0	$0	0
5	KSR 142.9 Manhattan-part	4	$4,000	1,899	$279.1	5	$2,504.4	40
6	KSR 137.3 St. George	0	$0	3,255	$484.9	0	$2,390.1	27
7	KSR 132.9 Wabaunsee	0	$0	1,970	$286.1	0	$1,428.5	16
8	KSR 126.3 (Wamego-part)	0	$0	2,683	$371.5	0	$2,405.2	24
9	KSR 117.7 (Wamego-part)	0	$0	5,117	$749.8	1	$934.2	44
10	KSR 110.2 Willard	8	$4,000	2,617	$371.5	78	$5,165.7	90
11	KSR 101.4	0	$0	845	$112.8	0	$328.5	4
12	KSR 96.1	0	$0	329	$44.4	0	$116.9	2
13	KSR 92.9	0	$0	389	$51.4	0	$356.7	4
14	KSR 84.4 Topeka-part	82	$82,000	335	$40.4	6	$1,048.5	10
15	KSR 83.3 Topeka-part	33	$33,000	8	$0.6	0	$80.4	6
16	KSR 74.5 Topeka-part	30	$30,000	1,457	$198.2	2	$4,023.1	59
17	KSR 68.4	0	$0	14	$2.0	0	$0	0
18	KSR 65.8	0	$0	2,594	$336.8	0	$1,109.9	19
19	KSR 64.6	0	$0	1,401	$194.4	0	$1,041.2	21
20	KSR 63.5 LeCompton	1	$500	767	$105.6	2	$217.7	8
21	KSR 61.2	0	$0	662	$93.6	0	$31.9	3
22	KSR 57.7	0	$0	1,500	$215.2	0	$548.7	9
23	KSR 53.5 Lawrence-part	1	$1,000	3,802	$533.3	0	$,1191.1	28
24	KSR 51.7 Lawrence-part	5	$5,000	27	$4.0	19	$1,873.5	19
25	KSR 37.0 Eudora	250	$125,000	6,414	$891.5	132	$15,016.5	272
Totals		1,150	$1,020,500	43,258	$6,068.2	981	$190,795.0	5,884

Sources: 1990 Census Tract Data, 1993 Land Cover, Kansas Applied Remote Sensing, Lawrence, KS, and Field Survey Data

Notes:
1. The 2000 Census data was unavailable for certain data fields required for our analysis. It is believed that the use of either census data set would result in no significant difference in the analysis results.
2. Any discrepancies are due to rounding.

b. <u>Description of Population at Risk</u>.

Population at risk (PAR) is defined as the persons that would be exposed to injury by floodwater if no measures were taken to evacuate. In this analysis, it includes the population in the area that would be inundated with a dam breach condition with a "sunny day" seismic event (at multipurpose pool level). The PAR includes people who reside, work, or conduct other activities in the area that would be flooded in the event of a dam breach.

The 2000 census data was not available in a proper format for this study, and thus, the population at risk was estimated using 1990 U.S. Census Bureau census tract (and census block when available) population and housing data for the reaches in the impact area. The flooded area outline for the breach condition, land use coverage, and the census tracts/blocks for the affected area were overlaid on the aerial mosaic map of the study area. Using a Geographic Information System program, the size of each census tract/block was measured and the total population in each block was recorded in the database. If the census tract/block did not completely lie in the inundated area, it was assumed that the population of the census tract/block was evenly distributed. Population in census tracts or blocks only partially within the inundated area was prorated by the GIS program based on the percentage of the census tract area within the flooded area. The inundated area includes portions of the communities of Manhattan, St. George, Wabaunsee, Wamego, Willard, Topeka, LeCompton, Lawrence, and Eudora. There is urban development in the cities of Manhattan, Topeka and Lawrence. The remaining portions of the inundated area are rural and agricultural in nature. The sizes of the communities in these areas generally reflect their agriculture nature.

The PAR could vary based on the transient population in the area, the season of year, and the time of day. Recreation opportunities will contribute to a transient population in the study area. Most visitors recreating below the dam are likely to come from the study area, and are not anticipated to add to the transient population. Although Manhattan, Kansas, in the area below Tuttle Creek Dam, is home to a large university population, a significant percentage of the student population takes summer classes, and many students stay year round in order to retain their off-campus rented housing. The total resident population in the study area is not expected to be likely to fluctuate significantly with the seasons. No adjustment was made to increase the PAR by the number of workers in the area. Some of the residential population in the floodplain would be at work, some in affected businesses, some would be elsewhere outside the floodplain. Some people who live outside the floodplain work in the floodplain. It is assumed that capturing these floodplain occupancy shifts would result in no significant change to the population at risk. No adjustment was made to the PAR to consider the time of day. Thus, for purposes of this analysis, the PAR is based on the estimated residential population in the inundated area. The population at risk is shown by reach in Table 7-2.

Table 7-2. Population at Risk within Reaches as Defined in Attachment B Maps.

Reach	Population at Risk
1	1,468
2	4,211
3	5,496
4	0
5	96
6	70
7	40
8	69
9	95
10	227
11	12
12	4
13	16
14	26
15	9
16	156
17	0
18	53
19	56
20	14
21	10
22	22
23	66
24	34
25	628
Total	12,878

c. Warning and Evacuation.

The threatened population is a subset of the PAR that is based on warning time and evacuation opportunities. It is defined as those persons exposed to floodwaters that remain after warning measures are taken. This includes persons that do not receive a warning to evacuate and do not receive sufficient warning time, are unable to evacuate, or choose not to evacuate after the warning is received.

Warning time is defined as the interval from when a public warning of a potential dam failure is initially disseminated until the arrival of the flood wave. Warning time is the most critical factor in reducing the threatened population and the potential for loss of life.

Tuttle Creek Lake has an Emergency Action Plan that outlines the steps the Corps personnel take to identify and respond to emergency conditions. Corps personnel at the project site monitor weather, river stages, and runoff forecasts, in addition to the structural integrity of the dam and its operating features. The Corps would initiate a public warning with adequate time for the local emergency management officials to disseminate the warning and begin evacuation procedures in the study area. It should be noted, however, that the general confusion and disruption of communications and transportation which would follow a major earthquake would certainly limit the effectiveness of warning and evacuation plans for downstream areas. The hazard map shown in Attachment B presents facilities that would present special challenges regarding response and evacuations.

For this study, the estimated warning time has been based on the time lapse between the seismic event and resulting initiation of the dam breach and the arrival of the breach flood wave and resulting bankful condition at the various locations along the river. These flood wave travel times are shown in Table 7-3 below.

Table 7-3. Travel Time in Hours after Earthquake, Bankful Condition and Maximum Stage Condition

Location	**River Mile**	**Hours after Earthquake**	
		Arrival Time-Bankful Stage	Peak Time – Maximum Stage
Kansas River, 8 miles upstream of the confluence with Big Blue River*	KSR 155.2	16	16
Manhattan-Big Blue River	BBR 5.3	5	11
Manhattan-Confluence Big Blue and Kansas Rivers	KSR 147.3	7	15
Wamego	KSR 126.3	15	19.5
Rossville	KSR 117.7	14.5	20.5
	KSR 92.9	26	26
Topeka	KSR 83.3	29	29
Lawrence	KSR 51.7	33	33

*Kansas River backwater, upstream of confluence of the Big Blue and Kansas Rivers

d. Probable Loss of Life.

Determination of the loss of life is based on the total population at risk, warning time, and evacuation. In the ideal situation, the total PAR would receive a warning with sufficient time to evacuate the flooded area and thus there would no loss of life. However, with a major earthquake in the area, the effectiveness of communication and warning systems and evacuation plans could be severely hampered and there would be a high risk for loss of life, particularly in the areas just below Tuttle Creek Dam. Additionally, in rural areas where the population is widely scattered, it is not possible that every single person would receive a warning. Additionally some would not heed the warning and would choose to remain in the flood-prone area. Even with adequate warning, loss of life could occur among this population.

(1) **Methodology.** The Corps of Engineers has no generally accepted method for determining the effectiveness of warning time to calculate loss of life. Professional judgment and the use of information from ongoing dam safety research are often used in making these estimates and assumptions. The Bureau of Reclamation (BOR) has recently published guidance entitled “A Procedure for Estimating Loss of Life Caused by Dam Failure”, DSO-99-06, September 1999. The BOR methodology is based on flood severity, amount of warning time, and the understanding of the severity of the flood, and was used in estimating loss of life for this study. Loss of life percentages were developed in accordance with the Bureaus guidance. Table 6 of the Bureau of Reclamation guidelines in “A Procedure for Estimating Loss of Life Caused by Dam Failure” DSO-99-06 was used for guidance. The table uses historical events and as a result some conditions do not have historical percentages. There were no cases that fit the high severity, more than 60 minutes condition that would occur in the high velocity portion of the flood plain in Manhattan Ks. In the medium severity category with over sixty minutes warning, the average fatality rate was 3.5% with a range of 0 to 8%. Time between the seismic event and inundation even in Manhattan KS was considerable and exceeded any of the tables time frames containing specific percentages for loss of life. Even though elapsed time is in hours however, significant loss of life will occur. First, as it is a seismic event there will be massive destruction of infrastructure such as roads, highways and utilities. Communications and even planned emergency procedures will be stressed, as there will be many problems to attend to. Some people will fail to evacuate even with warning and emergency personnel may also be in the area and at risk. Additionally a levee downstream from the dam, which protects a portion of Manhattan, would fail with a seismic event. As a result of these items LOL was estimated at 5% in the high velocity areas (near the river) and 3% in the lower velocity surrounding areas in Manhattan. These percentages may understate LOL due to the above-mentioned complications, however they are sufficient to demonstrate a great danger to human life exists with a dam failure. Downstream on the Kansas River the loss of life percentage was decreased (ranging from 2 to 1%) as infrastructure damage would be less, flood depths would be less, and information of upstream damage would be relayed to flood plain occupants.

(2) **Flood Severity.** Assumptions about flood severity were made for each reach in the study area. For the reaches on the Big Blue River and the reaches on the Kansas River in the vicinity of Manhattan, Kansas, it was assumed that these would have high velocities and high maximum depth in at least a portion of the reach. These areas have extensive residential and commercial/industrial development in the flood plain. Each reach was also divided into high risk and medium risk zones. An assumption of high risk was used for the area nearest the channel due to the high velocity and depth. An assumption of medium risk was used for the portion of the reach nearer the perimeter of the flooded area. For the reaches on the Kansas River below the city of Manhattan, once the floodwaters entered the Kansas River and moved downstream, depths and velocities would decrease. Thus for these reaches, a smaller percentage of the affected area in the reach was assumed to be in the high risk zone. For reaches further downstream, medium risk and low risk factors were used due to the lower flood depths, less velocity and longer warning times.

(3) **Warning Time and Understanding of Flood Severity.** The time lapse between the seismic event and the bankful condition was used as an estimate of warning time. The reach nearest the dam would have 5 hours between the seismic event occurrence and the bankful condition in the river. There is an emergency warning program in place, and five hours would be assumed to be adequate warning time to significantly reduce potential loss of life from flooding. However, with a major seismic event, there would be potentially significant earthquake damage. Communications could be disrupted, could be confusing or lacking altogether during the period immediately following a major earthquake. There could be difficulty in understanding the potential for flooding from an impending dam failure in conjunction with the earthquake. Transportation would likely be disrupted and evacuation routes could be difficult or unusable because of earthquake damage. Although warning time may appear to be adequate, because of the potential earthquake damage and other effects in the area, it was assumed that the effectiveness of downstream warning and evacuation plans would be severely limited. The hazard potential classification for loss of life is high because there is extensive residential and commercial/industrial development in the flood plain downstream of the project, particularly in the Manhattan, Kansas area. Based on this assumption, a 5 percent loss of life estimate was used for the population in the high-risk areas in the upstream reaches and 3 percent was used for the medium risk areas in these reaches. For the reaches located further downstream, the loss of life factors were gradually decreased to zero. Loss of life factors were decreased in these lower reaches because warning times are greater, there would be better understanding of the potential for flooding, flood depths and velocities would decrease as the wave moves downstream, and earthquake disruptions are expected to be of lesser magnitude in these lower reaches. Table 7-4 displays the potential loss of life estimates by reach.

Table 7-4. Potential Loss of Life (With Warning)

Reach (see Attachment B)	**Potential Loss of Life for Population in High Severity Area (high depth, high velocity)**	**Potential Loss of Life for Population in Medium Severity Area (moderate to low depths and velocities)**	**Potential Loss of Life- Total for Reach**
1	29	31	60
2	84	88	172
3	55	88	143
4	0	0	0
5	1	1	2
6	1	1	2
7	0	1	1
8	0	1	1
9	0	1	1
10	0	2	2
11	0	0	0
12	0	0	0
12	0	0	0
14	0	0	0

Reach (see Attachment B)	**Potential Loss of Life for Population in High Severity Area (high depth, high velocity)**	**Potential Loss of Life for Population in Medium Severity Area (moderate to low depths and velocities)**	**Potential Loss of Life- Total for Reach**
15	0	0	0
16	0	0	0
17	0	0	0
18	0	0	0
19	0	0	0
20	0	0	0
21	0	0	0
22	0	0	0
23	0	0	0
24	0	0	0
25	0	0	0
Total	170	214	384

e. Economic Losses With and Without Failure.

These are direct property losses due to flood-damaged homes, businesses, and infrastructure. A dam breach would flood the residential and commercial areas downstream, and would have many adverse economic consequences. Structures and contents would be damaged or destroyed. Public property such as roads and bridges would be damaged, potentially interrupting vehicular traffic. Recreation related businesses located above and below the dam would be severely disrupted. Residents' lives would be disrupted, with potentially traumatic and negative emotional experiences of personal loss and of dealing with flood clean-up requirements. Public health and relief needs of affected people would be another adverse impact.

Commercial and residential losses are based on the total investment value of structure and contents. Actual damages would be dependent upon the depth of floodwater and the extent of the damage relationship. In a dam breach situation, the intensity of flows and the rate of rise in floodwater would likely result in high percentage damage to much of the investment along the Big Blue River just below the project and in the upper Kansas River reaches in the vicinity of Manhattan, Kansas. As in the loss of life analysis, each reach in the study area was divided into high damage and medium to low damage zones. High damage zones in the reaches are those areas that would experience high velocities and major flood depths, and thus higher damages. These high damage zones represent a greater portion of the reaches along the Big Blue and the upper Kansas River reaches near Manhattan. A damage factor of 90 percent was used in the high damage zones in the reaches just below the project near Manhattan. The medium and low zones are those nearer to the flood perimeter in the upper reaches and factors lower than 90 percent were used. Once the floodwaters enter and move down the Kansas River, an even lower percentage of damage would be expected. As the flood wave moves through the reaches further downstream on the Kansas River, the size of the high damage zones were reduced or eliminated and the percent damage factor was also reduced to account for the decreased depths and decreased velocity. With adequate flood warning and no other complications from earthquake damage, residents and business owners might have time to move some of their high investment, highly damageable property to flood-free locations. However, for purposes of this analysis, no further adjustments were made for property removal.

Table 7-5 displays the estimated damage losses by category for each reach. These estimated damages do not include damages such as loss of the $56.2 million in average annual benefits of the project, negative environmental impacts, or the costs associated with destruction of the embankment itself.

Table 7-5. Economic Losses with Dam Failure.

Reach (see Attachment B)	**Losses With Failure**			
	Commercial	**Residential**	**Crop**	**Total**
1	$49,345,000	$30,728,000	$319,000	$80,392,000
2	$216,145,000	$73,620,000	$318,000	$290,083,000
3	$70,800,000	$4,032,000	$64,000	$74,895,000
4	$0	$0	$0	$0
5	$1,200,000	$1,002,000	$279,000	$2,481,000
6	$0	$4,302,000	$485,000	$4,787,000
7	$0	$514,000	$286,000	$800,000
8	$0	$649,000	$372,000	$1,021,000
9	$0	$168,000	$750,000	$918,000
10	$800,000	$1,033,000	$372,000	$2,205,000
11	$0	$66,000	$113,000	$179,000
12	$0	$23,000	$45,000	$68,000
13	$0	$71,000	$52,000	$123,000
14	$8,200,000	$105,000	$40,000	$8,345,000
15	$3,300,000	$8,000	$1,000	$3,309,000
16	$3,000,000	$402,000	$198,000	$3,600,000
17	$0	$0	$2,000	$2,000
18	$0	$111,000	$150,000	$261,000
19	$0	$104,000	$80,000	$184,000
20	$0	$11,000	$30,000	$41,000
21	$0	$2,000	$20,000	$22,000
22	$0	$27,000	$50,000	$77,000
23	$0	$60,000	$100,000	$160,000
24	$150,000	$56,000	$0	$206,000
25	$1,250,000	$300,000	$80,000	$1,630,000
Total	$338,290,000	$116,209,000	$3,453,000	$457,952,000

f. Environmental and Other Critical Losses.

Critical facilities in the inundated area in Manhattan, Kansas include a wastewater treatment plant, sewage disposal facility and municipal waterworks facility, with potential loss of water supply and sewer services. City of Manhattan administrative offices are located on the edge of the flooded area and could suffer some impact. A city police department and patrol division and one fire station are also located in the inundated area, thus potentially hampering provision of essential services. A key transportation link that would be severely impacted or likely destroyed is the U.S. Highway 24 Bridge over the Big Blue River. Major portions of Highway 24 would also be inundated and closed. Kansas Route 18 would be inundated and potentially closed for some period of time causing traffic disruption, and potentially delaying access to medical facilities. The airport is also located on Highway 18 West of Manhattan. The railroad bridge adjacent to the U.S. Highway 24 bridge over the Big Blue River would also be impacted or destroyed.

7-04. Current Average Annual Benefits.

a. General.

Tuttle Creek Lake is used for the purposes of flood control, recreation, navigation, water supply, water quality and fish and wildlife. Benefits presented for cost justification are benefits developed and used in the authorized project document. They were updated to current values for cost comparison. Annual benefits for Tuttle Creek are $56,177,000 (Oct. 2001 values). Flood control benefits of $46,930,000 represent about 84 percent of total benefits; recreation benefits are $6,588,000 and navigation benefits are $2,659,000. An economic value was not computed for water quality or fish and wildlife. To verify reasonableness of updated values, historical data were also analyzed. The purpose for using the historical data was to verify that benefits have been occurring at a level consistent with the benefits presented in the authorizing document. As historical data represents a specific range of years it is not expected that they would match the annual benefits but they could reflect the general nature of benefits.

b. Historical flood control benefits.

Damages prevented by Tuttle Creek from 1960 through 2001 are $3.946 billion. The most significant years of damage reduction are as follows:

1974	$234,128,000
1996	$280,965,000
1995	$696,783,000
1999	$881,364,000
1993	$1,250,128,000

Dividing the benefits for the 42 years of record yields an average flood control benefit of about $94 million per year for the project to date. This represents non-updated values; they would be greater if updated to 2001. Annual flood reduction benefits for the authorized project in 2001 values are $47 million. Authorized annual benefits developed for the project are used in the report as they are the approved benefits and reflect an average over time. Historic benefits are displayed to demonstrate that the benefits are reasonable.

c. Recreation.

Recreation benefits from the authorized project report updated to 2001 values are $6.6 million annually. Visitor hours for recreation from 1988 to 2000 are shown below. These figures are derived strictly from Corps of Engineers' sources. The Kansas Department of Wildlife and Parks indicates that they believe that the stated visitor hours and benefit value are both significantly understated.

Year	Visitor Hours
1988	2,582,423
1989	3,585,458
1990	3,078,970
1991	3,970,358
1992	2,870,450
1993	1,555,453
1994	2,482,685
1995	2,144,915
1996	2,583,486
1997	2,479,806
1998	2,427,236
1999	2,517,837
2000	2,831,148

d. Water Supply Contracts.

There are three water supply contracts entered into at three different dates. They are A: 1990 (27,500 acre feet) for $1,904,500; B: 1994 (8680 acre feet) for $650,000; and C: 1996 (13,850 acre feet) for $1090,400. O&M for the three contracts totaled $44,426 in 2001. Water supply benefits have not been quantified as part of the authorized project. The cost of storage for these contracts would represent a minimum estimate of current water supply benefits. These contracts would not be affected during project construction. Lake uses will remain unchanged during this period.

e. Loss of Life; Hydrologic Deficiency with PMF Event.

Given that the hydrologic deficiency requires only minor modifications performed in conjunction with seismic stabilization, the LOL analysis is limited in scope. Hydrologic modeling of the Probable Maximum Flood with and without dam failure in Manhattan, Kansas was performed. The downstream flood routing was performed in 1984 and was not reanalyzed for this report. This routing remains representative of the magnitude of the downstream conditions that would be present. The City owned Manhattan levee unit would be overtopped with the PMF both with and without dam failure. With dam failure, the flood depth increases from approximately 10 feet to approximately 21 feet in downtown Manhattan as shown on Plate 13 in Attachment B. The additional inundated area is over 130 square blocks of predominantly residential area but includes two schools, a fire station, and Manhattan City Hall. This increased inundated area would result in an increased PAR of about 2000 in a city of approximately 50,000. Travel time is short in this reach with the wave from the failure riding on top of floodwater already in the valley. The dam failure wave would be of medium severity in the Manhattan area with short warning time (less than one hour) resulting in 1 to 5 percent LOL = 20 to 100 fatalities. Additional damage and potential loss of life would occur downstream on the Kansas and Missouri Rivers. Plate 14 in Attachment B shows the limited additional inundated area in the Oakland and Belmont areas northeast of Topeka, Kansas. Plates 15 and 16 in Attachment B show the increased inundation in the Kansas City area. The failure of the dam during the spillway design flood results in the overtopping of the Kansas City levee system and inundation of massive areas including the Fairfax, Kansas City North, Harlem, east bottoms, and Birmingham areas that would not occur without dam failure. Economic damages in these downstream areas would be massive and additional LOL would be likely. Due to the limited costs of the embankment crest protection measures, the large inundated area in Manhattan, and the incremental LOL risk in Manhattan, quantitative determination of LOL and damages in areas further downstream is not deemed necessary to support the embankment crest protection.

7-05. Alternatives Considered for Seismic Retrofit.

a. General.

In accordance with the requirements of ER 1110-2-1155 the alternatives presented in Figure 7.2 were considered and evaluated in Phase II investigations:

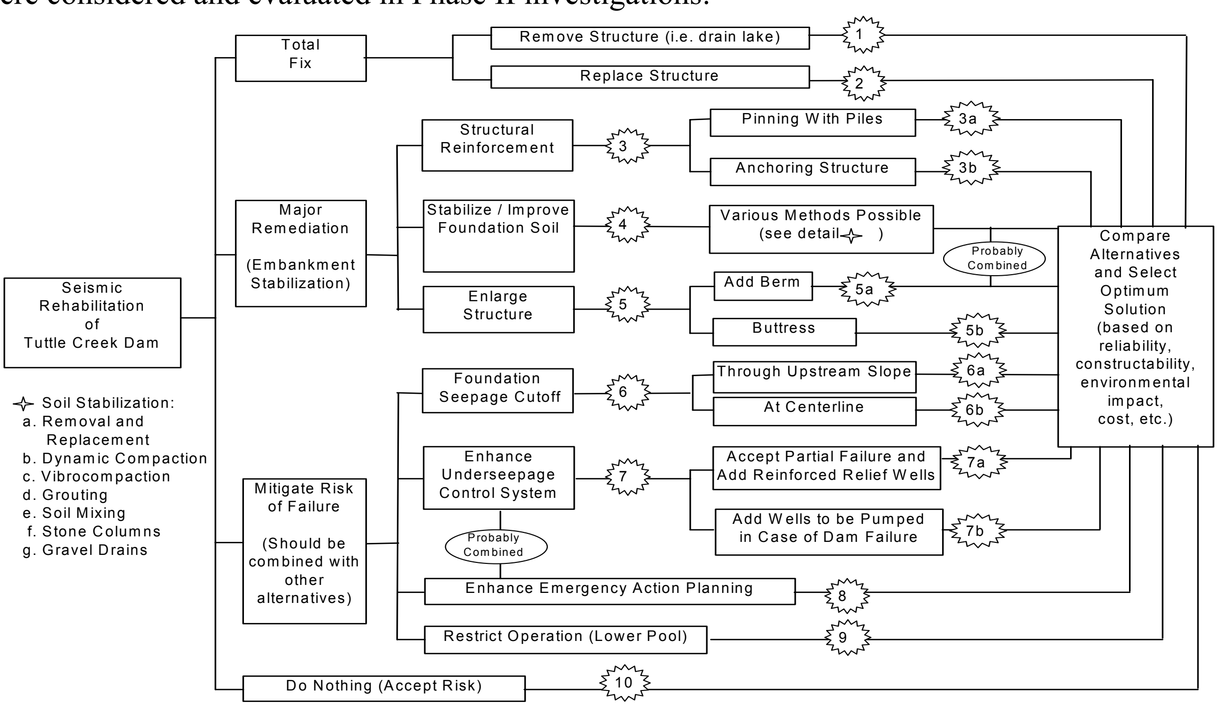


Fig. 7.2. Alternatives studied in Phase II investigations: the numbers 1 through 10 correspond to the alternatives detailed in the enclosed Phase II Special Investigations Report (Appendix VI)

For a detailed description of the alternatives refer to Appendix VI "Phase II Special Investigations". All alternatives were evaluated through a complex process, which included the following stages:

(1) **Initial screening**, based on the following acceptance criteria:

(a) **Safety Requirement**: in the event of MCE occurrence loss of life should be prevented. To quantify this requirement the following post-earthquake conditions have been defined (see Figure 7.3 for allowable deformations):

- factor of safety for post-earthquake limit equilibrium 1.2 or greater;
- maximum 5 feet lost of freeboard;
- maximum horizontal deformation of 1 foot at the downstream toe, and
- maximum horizontal deformation of 10 feet at the upstream toe.

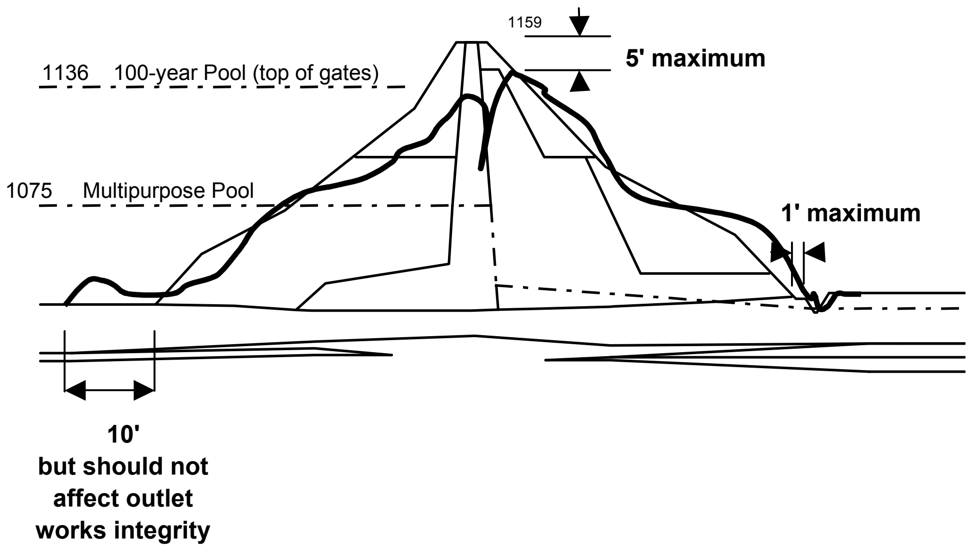


Fig. 7.3. Acceptable Deformations (not to scale).

(b) **Economic Requirement:** the annualized cost of modification should not exceed the annualizad project benefit. Annual benefits for the authorized plan at Tuttle Creek lake are $55 million. At the current government interest rate of 6 5/8%, rehabilitation costs up to $800 million could be justified.

(c) **Maintain Project Purpose:** Tuttle Creek lake was authorized for: recreation, water supply, fish & wildlife, flood control, water quality, and navigation. All these functions should be preserved unaltered after remediation.

(d) **Technical Feasibility:** Any alternative to be compared in view of selection of the rehabilitation solution should be feasible under standard construction procedures, its results should be verifiable, should be safe during construction, and the remediation should not create a new defect.

(2) **Value Engineering (VE) Study**. A panel of four geotechnical engineers, a cost estimator, an environmental planner, and an economist evaluated every potential alternative and established a ranking based on the following criteria:

(a) **Limit deformation**: the deformations should be small enough to prevent loss of lives following uncontrolled releases, which can be due to: (1) overtopping if a flood event occurs soon after the damaging earthquake, (2) severe damaging of the upstream impervious blanket, (3) damaging of the pressure relief system, or (4) major cracking that may lead to failure by erosion/piping.

(b) **Unacceptable impact**: any remediation action should improve the performance of the structure under all loading conditions.

(c) **Maintains authorized project purposes and benefits**: flood control, recreation, fish and wildlife, water quality and supply, navigation on the Missouri River.

(d) **Cost**: should be reasonable as compared to quantifiable benefits. The initial cost of remediation should be minimized. Negative long term economic impact of modifications should also be minimized.

(e) **Constructability**: includes safety during construction of people downstream, compatibility of the construction method with the site, ease of construction and acceptable sequencing and duration of construction.

(f) **Verifiability of results**: ease of analysis, clear criteria for establishing the treatment effectiveness.

(g) **Environmental**: the impact on the environment should be acceptable for both short and long term; general public acceptance is required.

The results of the VE study are presented in detail in Appendix IV.

(3) **Final Evaluation**. The results of the initial screening and of the VE study were considered, as well as the recommendations of a panel of experts: Professor W. Liam Finn from University of Kagawa, Japan (formerly with University of British Columbia, Canada), Dr. Gonzalo Castro from GEI Consultants Inc., Winchester, Massachusetts, and Dr. Mary Ellen Hynes from the Engineer Research and Development Center, Vicksburg, Mississippi. The best rated and recommended alternatives were studied in detail for all the criteria listed above.

b. Brief Presentation of Alternatives.

A. **No Action**.

(1) The probability of occurrence of a seismic event capable to liquefy the dam foundation and, consequently, to induce major deformations of the dam and uncontrolled releases, is remote. However, a lower seismic event (the threshold earthquake) may induce liquefaction underneath the downstream slope and, consequently, failure of the lower portion of the slope. Such a failure would fracture the existing relief wells and create piping potential that can trigger dam failure. The threshold event has a return period of about 1,800 years. Loss of life is very probable if the embankment fails by piping.

(2) With dam failure, there would be significant impacts to the residents and users of the land and development resources downstream of the dam. The annual flood control and other benefits provided by the project would be lost and there would be additional significant downstream economic damages that would occur with a dam breach and flood wave. There would be high risk potential for loss of life in the upstream reaches below the dam. Emergency services would be impacted due to impacts to access routes and transportation infrastructure. Loss of critical services would occur, including loss of water and sewer services in the upstream reaches, and there would be environmental damages and losses. With dam failure, there would also be future costs to the Federal Government. In addition to the high cost of repairing or rebuilding the dam after failure, there could be significant costs to settle legal claims. These would include the time and resources spent in settlement negotiations and potential litigation if the Government does nothing to correct the problems identified. The dollar amount of actual claim settlements and litigation damage payouts would be significant.

(3) The data base on earthquake recurrence in the area which may affect the dam is small, so that the risk analysis is not reliable. Therefore, there is a high degree of uncertainty in the evaluation of consequences of the possible failure, so that accepting the risk of failure is not an option.

B. **Partial Correction**.

Five variants of this alternative were analyzed:

(1) **Foundation Seepage Cutoff**. Positive control of underseepage would eliminate the necessity of pressure relief systems along the downstream toe and, therefore, the danger of piping if the existing system is destroyed by large deformations of the embankment near the downstream toe (the condition of maximum one foot horizontal deformation at the downstream toe may be relaxed). Two variants of this alternative may be effective in the conditions at Tuttle Creek Dam:

(a) **Cutoff through the upstream slope**, within the limits of the upstream impervious fill. This location minimizes the thickness of the existing fill that should be penetrated and does not require temporary lowering the pool. As seismic deformations are considered possible at this location, the allowable deformations should be coordinated with the thickness and flexibility of the cutoff wall.

(b) **Cutoff through central core**. The advantage of this location is that no significant seismic deformations are probable. Therefore both cement/bentonite backfills or concrete diaphragm walls are possible options, and their thicknesses may be minimized, within the limits of constructability.

A deep channel exists in the bedrock at approximately the middle of the valley so that a maximum depth of about 230 feet from the dam crest is necessary for positive cutoff. Taking into account that this alternative does not prevent liquefaction of foundation soil and embankment seismic deformation, and requires very deep cutoff walls, it was eliminated as a sole method of foundation improvement during the initial screening. However, it can be used in conjunction with other alternatives; specifically, to construct an upstream cutoff with jet grouting or deep soil mixing.

(2) **Enhanced Underseepage Control System** with two different options:

(a) Accept partial failure and **add reinforced relief wells**; Fifteen reliable wells are needed to prevent piping if MCE occurs with the lake pool at multipurpose level.

(b) Accept partial failure and **add wells to be pumped**. In this variant 13 additional wells would be installed 600 feet downstream from the toe, far enough to prevent damage to them if the downstream slope of the dam fails. They would not have any role in normal conditions. If some of the existing relief wells fail, their function may be taken by pumping from distant wells. A number of submersible pumps and electric generators should be operable at any time.

(3) **Enhanced Emergency Action Planning**. Failure of the dam would be accepted but measures taken to evacuate the population downstream before the releases can reach them. (Example: Santee North Dam, South Carolina.) Due to populated areas immediately downstream of the dam, any evacuation plan would not be feasible; not meeting the safety requirement, this alternative was eliminated by the initial screening.

(4) **Restricted Lake Operation** (permanently lowering normal pool). Although the existing freeboard (based on the multipurpose pool elevation of 1075 feet m.s.l.) is 84 feet, this is not sufficient, as large deformations and severe cracking are expected in the assumption of MCE occurrence. Prevention of failure by piping, if the relief pressure system becomes non-functional following large deformations of the dam, requires permanent lowering of the lake level by approximately 25 feet (to a normal level of 1050 feet m.s.l., see Figure 7.4; justification of this limiting pool is provided in Appendix VI, pages 7-34 and 35). Such a dramatic pool level reduction would result in essentially a dry flood retention structure.

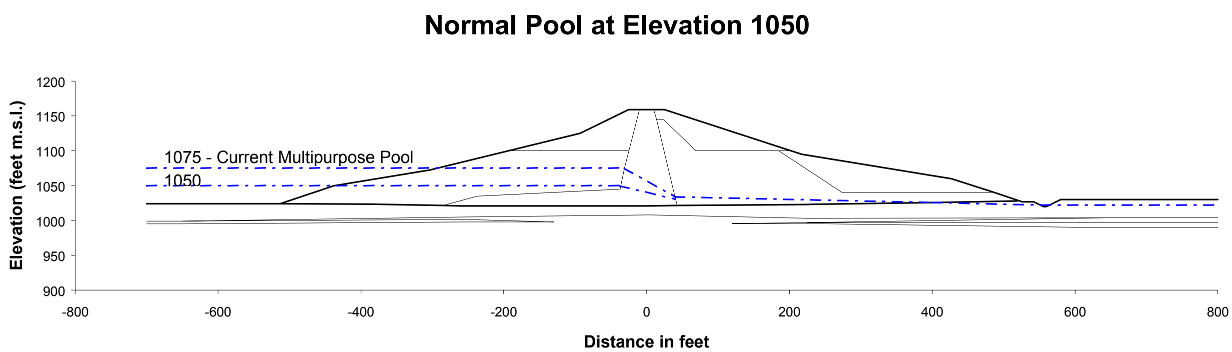


Fig. 7.4. Conceptual sketch of the alternative of permanently lowering normal pool.

The remaining storage amount would not be sufficient to provide dependable yield for navigation, water supply and water quality project purposes. Recreation, fish and wildlife would also be severely impacted due to the change in pool. Therefore, lowering of the pool elevation would adversely impact numerous project purposes and require a reallocation of the project and Congressional approval. This alternative does not meet the initial screening requirement of no change in project purposes, so it was eliminated.

(5) **Enhanced Drainage Capacity.** This alternative is intended to significantly improve the ability to drain the lake in the event of embankment failure, following a strong earthquake. Features of this alternative are discussed in some detail in the companion Environmental Impact Statement (paragraph 2.1.4) and will not be included here. This alternative was removed from further detailed consideration due to the high construction uncertainty and risks, high cost to construct and maintain, failure to eliminate downstream flooding and potential loss of human life and property, and requiring human intervention to operate after a seismic event.

C. **Complete Correction**.

(1) **Reinforce Embankment**. This solution is intended to preserve the dam almost intact, even if layers of the foundation soil liquefy due to the seismic action. Minor settlement and cracking of the embankment are possible, as liquefied soil is free to flow from underneath the dam. Two alternatives were evaluated under this category:

(a) **Reinforce embankment with piles**. Concrete piles would be used to pin the lower portion of the slope into the stable foundation, underneath the liquefiable layers. On the upstream side it would be necessary to drill through the embankment fill (where big stones are expected) and to drive the piles into the foundation soil. On the downstream side temporary excavation of the existing berm would be needed. It may be necessary to build a plant for manufacturing of piles in the vicinity of the dam. This alternative is presented in Figure 7.5.

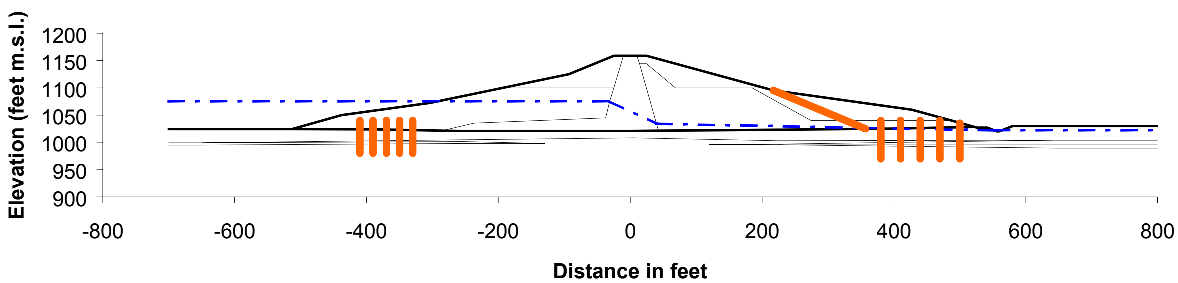


Fig. 7.5. Reinforcing embankment with piles.

The preliminary design of this alternative used as a comparison the remediation applied at Sardis Dam, Mississippi. At Tuttle Creek the thickness of the liquefiable layer is about three times greater than at Sardis Dam; this would require roughly three times more piles per unit length of slope or considerably stronger piles. These options were considered not technically feasible, so that the alternative of reinforcing with piles was eliminated.

(b) **Reinforce embankment with anchors**. High capacity anchors encased in concrete can prevent excessive deformation. Concrete cracking may be prevented by pre-tensioning the anchors. The forces in anchors should be distributed into the embankment fill through a reinforced slab on the slope surface. See Figure 7.6 for the general configuration.

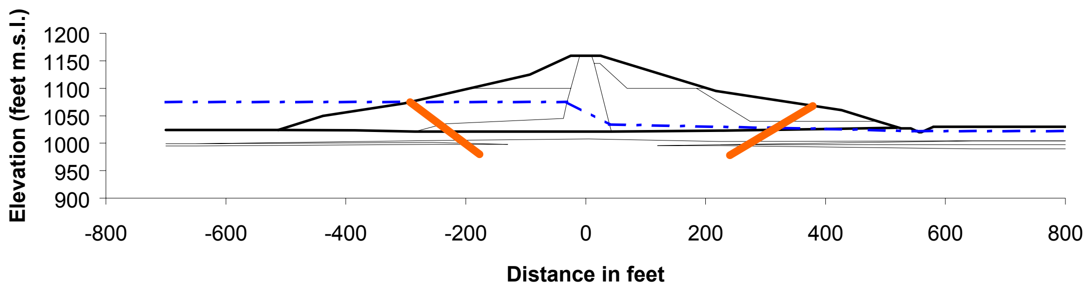


Fig. 7.6. Reinforcing embankment with anchors.

For both the upstream slope and downstream slope the numbers and lengths of required anchors was uneconomical as compared with the solution using piles. Therefore, this alternative was considered not technically feasible and was not studied in detail.

(2) **Stabilize Foundation Soil**. On the upstream side of the dam the stabilization equipment should operate from a platform through holes predrilled within the shale and limestone fill. On the downstream side an alternate option would be to temporarily remove the existing berm fill. The concept of this alternative is presented in Figure 7.7.

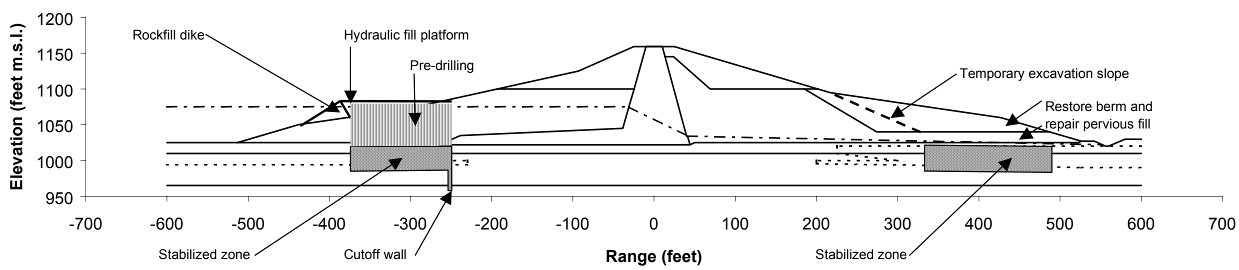


Fig. 7.7. Conceptual sketch of foundation soil stabilization.

The final decision on the selected stabilization method should be based on the results of full scale test sections at the dam site. Figure 7.8 presents the methods previously used for seismic liquefaction mitigation in the United States; the gradation range where the various methods are efficiently applicable is compared with the gradation ranges of the problem soils encountered in the Tuttle Creek dam foundation. Other more recently developed stabilization methods may also be effective (jet grouting, grout piles, etc.).

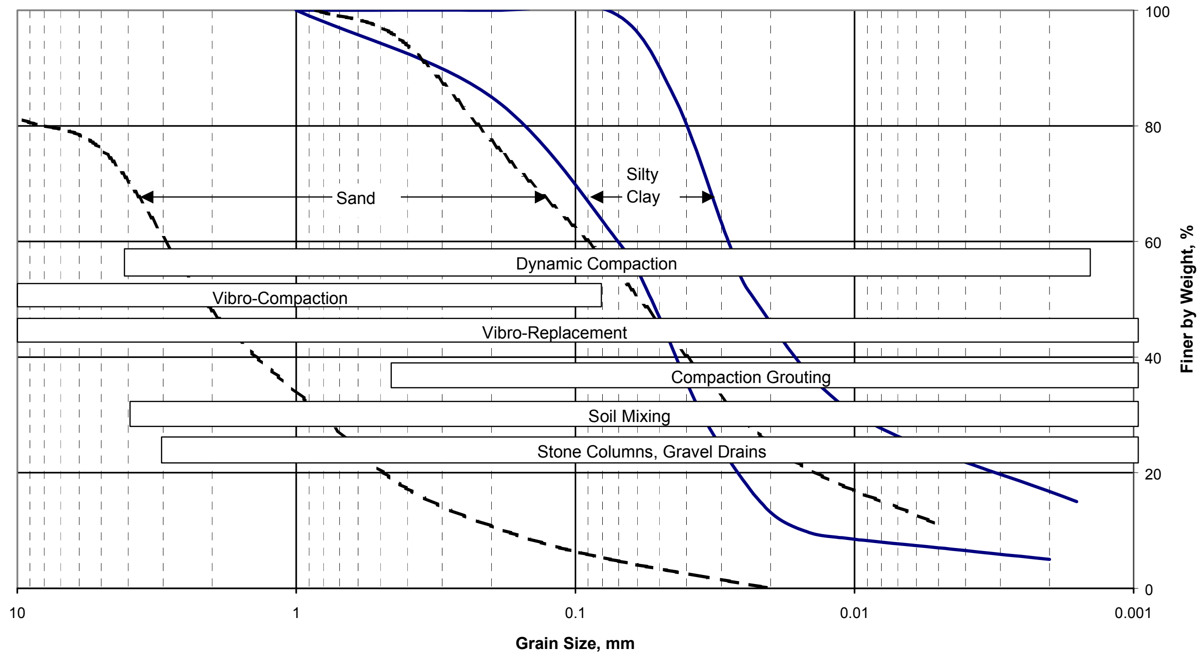


Fig. 7.8. Gradation range of problem soils and efficiency of various stabilization methods (Mitchell et al., 1998).

The problem soils include:

- Liquefiable sand, a layer about 16 feet deep in the foundation soil (located between elevations 1010 and 994) under the lower half of the upstream slope and upstream of the dam;

- Liquefiable sand, a layer about 20 feet deep in the foundation soil (located between elevations 1010 and 990) under the lower half of the downstream slope and downstream of the dam;

- Silty clay and clayey silt in the foundation blanket, where in contact with the liquefiable sand, were assumed to be susceptible to significant strength loss due to pore pressure increase and large deformations.

The following methods of foundation soil stabilization were evaluated.

(a) **Removal and replacement of liquefiable material**. In the case of Tuttle Creek Dam, deep excavation, on the order of 30-40 feet, is necessary if all problem soil is to be removed. The excavated material can be replaced, becoming non-liquefiable if properly compacted. At the downstream toe the water table is normally at a depth of 7-8 feet, so an excavation to this depth would require temporarily lowering the reservoir and a dewatering system that may include the existing wells. The removal and replacement may be restricted to the upper zone with cohesive soils (15-20 feet in depth) with in situ stabilization of the sand underneath. Removal and replacement is unacceptable upstream, where even temporary complete lowering of the pool is not a desirable option.

(b) **Dynamic compaction** (heavy tamping). Dynamic compaction is a competitive solution from cost and efficiency points of view, but it has restricted applicability at Tuttle Creek dam. The method is efficient only if applied at the surface of the soil to be improved or on a structural fill of selected material and relatively small thickness (sand blanket with thickness of the order of 5 feet); it is, therefore, not applicable under the upstream slope and requires temporarily removal of most of berm fill for stabilization of soil under the downstream slope.

(c) Densification by **vibrocompaction**. This method is considered "not feasible" in the case of Tuttle Creek dam because of lack of efficiency in fine grade materials (blanket and upper portion of aquifer).

(d) **Jet grouting**. Jet grout segments can be used to create zones of containment of the liquefiable soil. While not reducing the risk of liquefaction, containment minimizes the potential for catastrophic failure by preventing the flow of the liquefied soil. In addition, the grouted zones have increased shear strength, which opposes deformation and improves stability. Jet grouting is considered an ideal solution for the upstream slope. A full depth jet grouted wall would assist in controlling underseepage. It also increases the strength and decreases the permeability of the foundation soil underneath the upstream slope.

(e) **Soil mixing**. The deep soil mixing method can be used to install a wall along the downstream toe, to prevent flow of the liquefied soil from under the structure. The high-productivity specialized equipment cannot work through pre-drilled holes, so that the method is not applicable to the upstream slope. However, soil mixing with Portland cement is considered the best solution for the downstream slope stabilization.

(f) Densification by **stone columns**. There are various methods of stone columns construction, basically classified in two main categories: (1) the wet (vibro-replacement) installation method and (2) dry bottom feed stone columns. Densification is the primary mechanism of treatment, with drainage being a secondary benefit. However, increasing permeability under the upstream slope following the perforation of the impervious blanket may be detrimental for long term seepage and stability, so it should be applied only in conjunction with a positive cutoff downstream of the stabilized zone. Downstream, relief of seepage pore pressure during normal operation conditions may induce internal erosion. Either a positive cutoff upstream is necessary, or the dry bottom feed method with selected gravelly material should be selected.

(g) **Gravel drains**. The difference between gravel drains/ piles and stone columns is mainly the technology used for installation. Gravel drains/piles may be installed with impact driven casing method (Franki) or the vibro-replacement method. The Franki method is preferred as more effective in the cohesive materials of the blanket.

(3) **Enlarge Embankment**. Enlargement at the base is done by building berms either upstream or downstream, or both, using mostly dredged material from reservoir or the lake downstream. Enlargement at the crest level by construction of a buttress, increases the width of the structure at the retention level and prevents piping even if significant cracking and displacements occur in the embankment fill.

(a) Build **berm upstream**. The berm upstream should be built underwater. The top of the berm will be above multipurpose pool and will create a dry platform in normal conditions from where the soil underneath can be improved. Alternatively, the soil improvement may be performed before building the berm. The stabilizing effect of the berm is significantly decreased by submergence. Also it was assumed that the liquefiable sand extends indefinitely upstream, which is a legitimate assumption. Therefore, a relatively wide berm of 400 feet is necessary (see Figure 7.9).

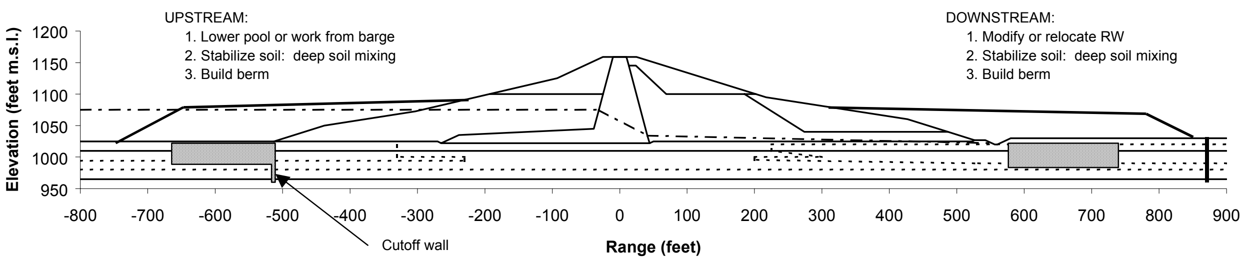


Fig. 7.9. Berms on stabilized soil.

(b) Build **berm downstream**. For the downstream slope the necessary width of the berm is 425 feet. Stabilization of soil underneath is necessary to prevent damage to the pressure relief system if the berm fails. The existing pressure relief system must remain functional or should be replaced with a new system (see Figure 7.9).

(c) Add **buttress** downstream. The preliminary design determined the need of a buttress 100 feet wide at the crest and 300 feet wide at the ground level. The upper portion should be reinforced and a strong internal drainage must be built between the new and the old embankment fill. A new pressure relief system is recommended. Soil improvement under the buttress is necessary. Figure 7.10 presents this variant as it was optimized during the Value Engineering study process.

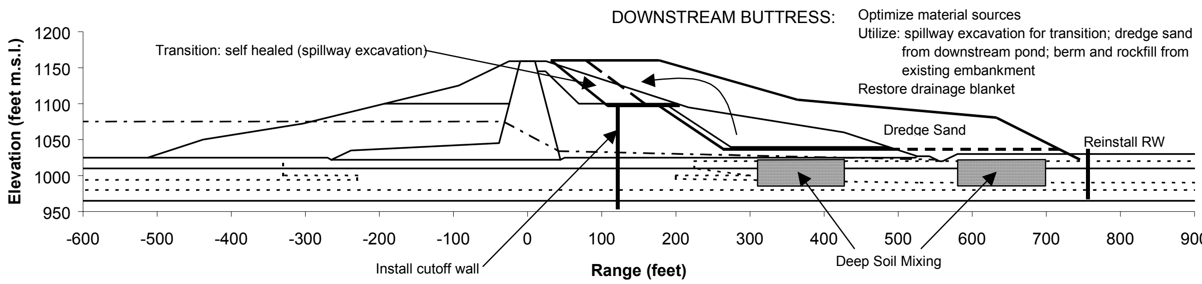


Fig. 7.10. Preliminary design of the buttress.

D. **Breach Embankment**. The breach should be wide enough (approximately 500 feet at the bottom) to leave safe passage to water from a major flood event. The remaining fill should be protected against erosion. The outlet works and portions of the embankment dam, although no longer necessary, may remain in place. Figure 7.11 presents the general concept of this alternative.

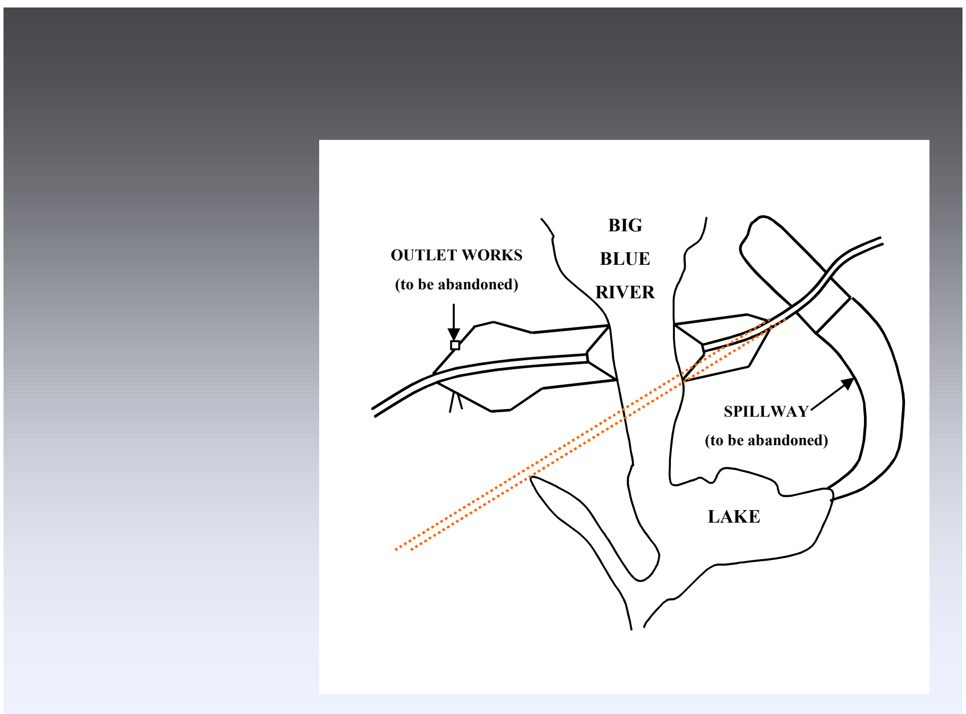


Fig. 7.11. Remove part of the dam (drain lake).

Although technically sound, this alternative was rejected because two of the four initial screening requirements are not met: the annual project benefit would be completely lost; and all authorized project functions (recreation, water supply, fish & wildlife, flood control, water quality, and navigation) will not be maintained. In addition, the environmental impacts would be significant and not acceptable to the public.

E. **Replace Embankment**. The replacing embankment should be a dam with the same height and similar features as the existing dam. The foundation soil underneath should be stabilized; however, the dam built on stabilized soil may have significantly steeper slopes than the original embankment. Relief wells or a positive cut-off are also needed. If built immediately downstream of the existing structure, the new dam may use the existing spillway and outlet works (see Figure 7.12).

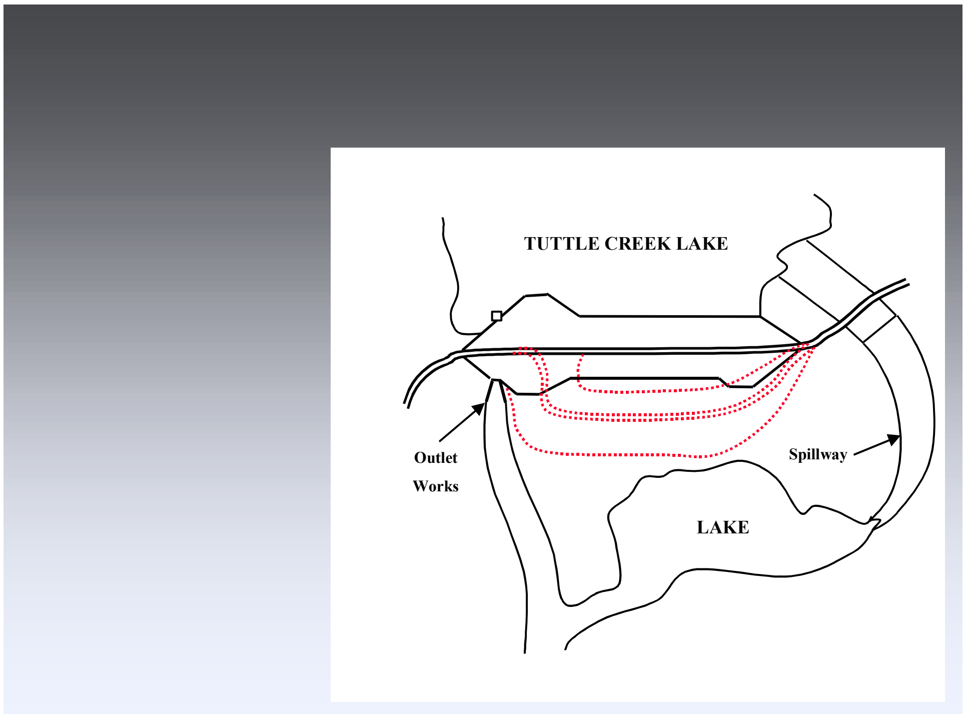


Fig. 7.12. Conceptual sketch of the structure replacement.

Although the preliminary cost estimate is less than the threshold value for not meeting the economic initial screening criterion, the annualized cost significantly exceeds the annualized cost of most other alternatives. This alternative would also generate severe environmental impacts.

c. Results of initial screening.

Table 7.6 summarizes the results of the initial screening that considered the acceptance criteria listed at 7.05.a(1).

Table 7.6. Summary of the initial screening results.

Alternative / Variant	Criteria Met (Y = Yes; **N** = No)					Comment
	No loss of life for MCE	Annualized cost less than project benefit	No change in project purposes	Technically feasible		
				U/S	D/S	
A - No action	**N**	Y	Y	Y	Y	Eliminated
B(1) - Foundation seepage cutoff:						
(a) - through u/s slope	**N**	Y	Y	Y	N/A	Eliminated*
(b) - through central core	**N**	Y	Y	Y	N/A	Eliminated*
B(2) - Enhanced underseepage control system:						
(a) - add reinforced wells	**N**	Y	Y	N/A	Y	Eliminated*
(b) - add wells to be pumped	**N**	Y	Y	N/A	Y	Eliminated*
B(3) - Enhanced emergency action planning	**N**	Y	Y	Y	Y	Eliminated
B(4) - Restricted lake operation	Y	Y	**N**	Y	Y	Eliminated
B(5) - Enhanced drainage capacity	**N**	**N**	Y	Y	Y	Eliminated
C(1) - Reinforce embankment:						
(a) - with piles	Y	Y	Y	**N**	**N**	Eliminated
(b) - with anchors	Y	?	Y	**N**	**N**	Eliminated
C(2) - Stabilize foundation soil:						
(a) - remove and replace	Y	?	Y	**N**	Y	OK d/s only
(b) - dynamic compaction	Y	Y	Y	**N**	**N**	Eliminated
(c) - vibrocompaction	Y	Y	Y	**N**	**N**	Eliminated
(d) - grouting	Y	Y	Y	Y	Y	OK
(e) - soil mixing	Y	Y	Y	Y	Y	OK
(f) - stone columns	Y	Y	Y	Y	Y	OK d/s only
(g) - gravel drains	Y	Y	Y	Y	Y	OK d/s only
C(3) - Enlarge embankment:						
(a) - build berms	Y	Y	Y	Y	Y	OK
(b) - add buttress	**N**	Y	Y	**N**	Y	Eliminated*
D - Breach embankment	Y	**N**	**N**	Y	Y	Eliminated
E - Replace embankment	Y	**N**	Y	**N**	Y	Eliminated

Note: * Although eliminated, this variant may be used in conjunction with other alternatives.

d. Conclusions of the Value Engineering study.

The results of the VE study are presented in detail in Appendix IV.

(1) **Rating criteria**. Twenty alternatives were identified and evaluated. The following criteria were used in evaluation:

Description	Weight(%)
Limits deformations	24
Unacceptable impacts	16
Maintains project purposes	16
Cost	12
Constructability	12
Verifiability of results	8
Environmental	12

Each criterion was rated for every alternative by each participant, using a scale from 1 to 5, as follows:

Rate	Rating Scale
1	Poor
2	Fair
3	Good
4	Very good
5	Excellent

(2) The five **best rated alternatives** (weighted rating between 3.85 and 3.60) are listed in Table 7.7. These five options are presented graphically in Figure 7.13.

Table 7.7. Selected Options.

<table>
<tr><th>Option No.</th><th>Treatment of Upstream Slope</th><th>Treatment of Downstream Slope</th></tr>
<tr><td>1</td><td rowspan="2">• Lower the pool temporarily (the goal will be to maintain the pool at el. 1060 during construction).
• Create a work platform at el. 1070 by cut and fill. There is 80% chance the platform will be flooded during construction, when the equipment should be evacuated from the work zone and the activity interrupted.
• Pre-drill pilot holes through limestone and shale embankment backfill.
• Stabilize by jet grouting the foundation soil between elevations 1022 and 992 (12' of cohesive soil and 18' of liquefiable sand).
• The two furthest d/s rows should be extended down to rock (average el. 960) to create a cutoff wall.
• Restore the original slope surface using mostly on site material.</td><td>• Excavate temporarily most of the berm and pervious drain underneath, down to the original ground surface (moving slot method, 500' to 800' wide).
• Stabilize foundation soil between elevations 1025 and 990 (15' of cohesive soil and 20' of liquefiable sand) using deep soil mixing. Create transverse walls in buttress, connected at their u/s end with a longitudinal wall.
• Restore pervious drain and berm.</td></tr>
<tr><td>2</td><td>• Excavate temporarily a small portion of the berm toe and pervious drain underneath, down to the original ground surface (moving slot method, 500' to 800' wide).
• Stabilize foundation soil between elevations 1025 and 983 (15' of cohesive soil and 27' of liquefiable sand) using jet grout by battering. Try to create transverse walls in buttress, connected at their u/s end with a longitudinal wall 15' deep (extended through the cohesive soil blanket only).
• Restore toe and pervious drain.</td></tr>
<tr><td>3</td><td rowspan="3">• Maintain the pool at multipurpose level, elevation 1075.
• Create a work platform with imported material at el. 1087, with platform toe a rockfill dike and the reminder of the platform dredged sand. There is 50% chance the platform will be flooded during construction, when the equipment should be evacuated from the work zone and the activity interrupted for the duration of the flood event (assumed about one month).
• Drill through platform to stabilize by jet grouting the foundation soil between elevations 1022 and 992 (12' of cohesive soil and 18' of liquefiable sand).
• The two furthest d/s rows should be extended down to rock (average el. 960) to create a cutoff wall.
• Finalize the work platform as a zone accessible to public, for recreation.</td><td>• Excavate temporarily most of the berm and pervious drain underneath, down to the original ground surface (moving slot method, 500' to 800' wide).
• Install gravel drains between elevations 1025 and 990 (15' of cohesive soil and 20' of liquefiable sand). Use the stone column technology with filling material adequate for preventing piping.
• Restore pervious drain and berm.
• Seed and mulch d/s slope.</td></tr>
<tr><td>4</td><td>• Pre-drill through rockfill to original ground surface.
• Install gravel drains between elevations 1025 and 990 (15' of cohesive soil and 20' of liquefiable sand). Use the stone column technology with filling material adequate for preventing piping.
• Restore pervious drain and berm.</td></tr>
<tr><td>5</td><td>• Build work platform using dredged material.
• Pre-drill and install cased pilot holes through berm.
• Stabilize foundation soil between elevations 1025 and 983 (15' of cohesive soil and 27' of liquefiable sand) using jet grouting. Create transverse walls in buttress, connected at their u/s end with a longitudinal wall.
• Remove most of the work platform to facilitate drainage of runoff.
• Seed and mulch d/s slope.</td></tr>
</table>

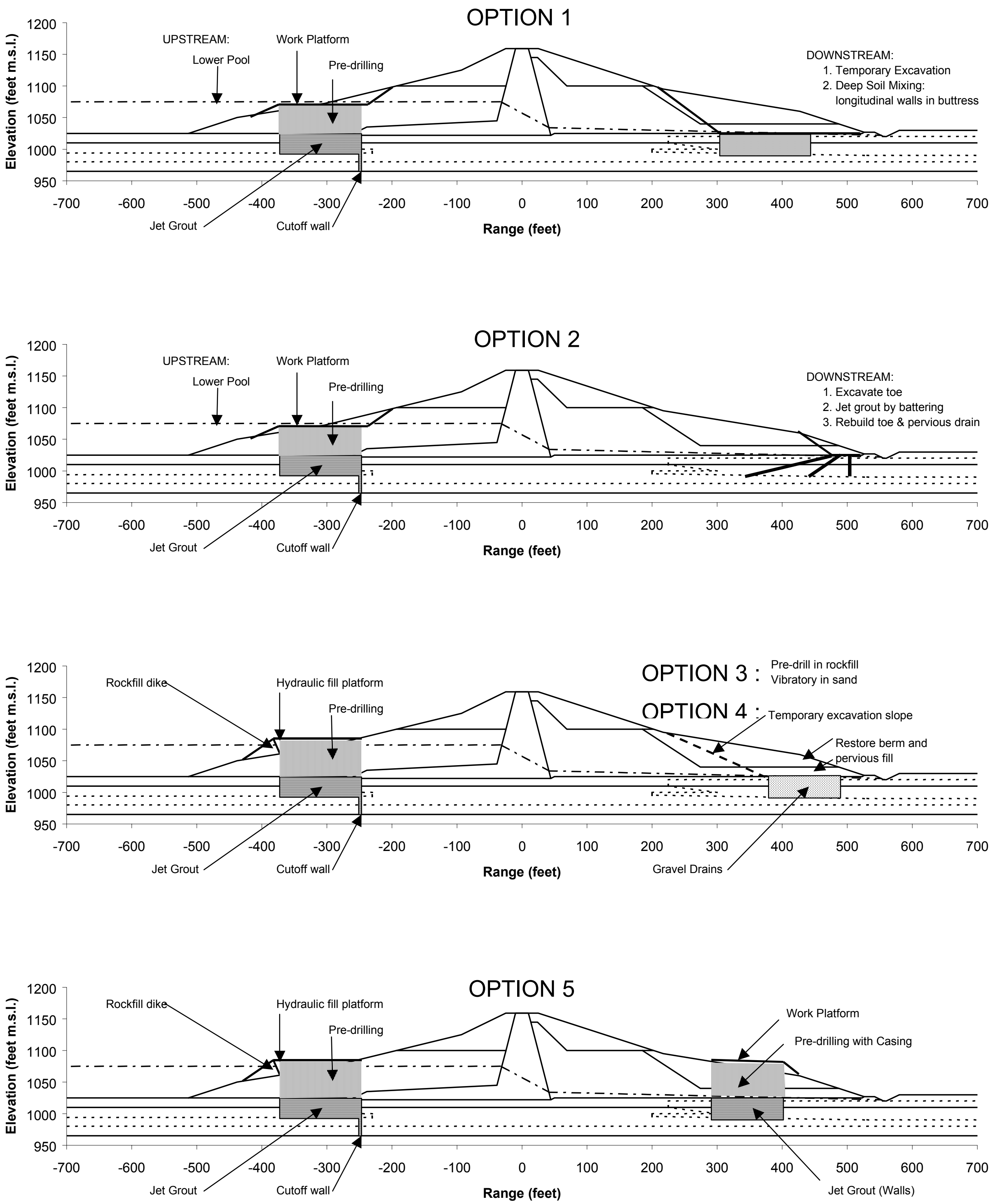


Fig. 7.13. Options selected by the Value Engineering study.

e. <u>Cost evaluation</u>.

Detailed cost analyses are presented in Appendix VI "Phase II Special Investigations - Part 2: Detailed Field Investigation and Evaluation of Repair Alternatives" (sub-chapter 7.6 and Appendix I). The following table summarizes the estimated costs of alternatives found acceptable from technical and environmental impact points of view.

Table 7.8.

<table>
<tr><th>Alternative Number</th><th>Description</th><th>Construction Cost
(Escalation to October 2001)</th></tr>
<tr><td>A</td><td>No action</td><td>No construction cost before strong earthquake occurrence</td></tr>
<tr><td>B(1)</td><td>Foundation seepage cutoff (in conjunction with jet grouting stabilization of soil under upstream slope)</td><td>$ 80,000,000 - $ 45,300,000
= $ 34,700,000</td></tr>
<tr><td>B(2)</td><td>Enhanced underseepage control system</td><td>Not evaluated: does not prevent failure and loss of life</td></tr>
<tr><td>B(3)</td><td>Enhanced emergency action planning</td><td>Not evaluated: does not prevent failure and loss of life</td></tr>
<tr><td>B(4)</td><td>Restricted lake operation</td><td>Minor construction cost</td></tr>
<tr><td>B(5)</td><td>Enhanced drainage capacity</td><td>Over one billion dollars</td></tr>
<tr><td>C(1)</td><td>Reinforce embankment</td><td>Not evaluated: not technically feasible</td></tr>
<tr><td>C(2)</td><td>Stabilize foundation soil:
- Under upstream slope:
• Jet grouting stabilization
• Including cutoff wall
- Under downstream slope:
• Jet grouting stabilization:
▫ After pre-drilling
▫ After temporary excavating berm
• Deep soil mixing (after temporary excavating berm)
• Gravel drains (stone columns)</td><td>

$ 45,300,000
$ 80,000,000

$ 137,500,000
96,100,000

$ 79,800,000
$ 59,850,000</td></tr>
<tr><td>C(3)</td><td>Enlarge embankment with berms:
- Upstream side (followed by soil stabilization)
- Downstream side (after soil stabilization)</td><td>
$ 70,300,000
$ 49,500,000</td></tr>
<tr><td>D</td><td>Breach embankment:
- With bypass highway
- With bridge</td><td>
$ 33,850,000
$ 35,150,000</td></tr>
<tr><td>E</td><td>Replace embankment</td><td>$ 405,000,000</td></tr>
</table>

f. <u>Final evaluation</u>.

It is considered that the best alternative for stabilization of the upstream slope is jet grouting from a platform built on the lower portion of the slope. The construction cost of this alternative is less than applying jet grouting from a berm built in the reservoir ($45.3 million compared with $70.3 million) and it implies much less adverse environmental impact. Installing an underseepage cutoff is also recommended. The additional cost of $34.7 million is justified by the possibility of relaxing the requirement of 1 foot deformation at the downstream toe, as the relief wells would not be critical for dam stability any more, even if the cutoff wall were not perfect.

The recommended alternative for stabilization of the downstream slope is deep soil mixing from a platform at elevation 1040, obtained by excavating the berm material ($79.8 million). This alternative minimizes environmental impacts due to the need for large quantities of borrow material, eliminates the permanent direct impact of a berm in the downstream state park, and minimizes temporary impacts. The temporary downstream slope of the portion of the dam where the berm will be excavated will be 1:2.75. The contamination with grout of the drainage blanket material (between elevations 1040 and 1025) will be prevented by lining the holes within it. Stabilization of the zone immediately adjacent to the relief wells is necessary, from the service road along the toe. It is noted that the active relief wells are located downstream of the collector ditch; the old relief wells located near the stabilization zone have been abandoned, although they are still functional.

The estimated total construction cost of the seismic remediation of the embankment is, therefore: $45,300,000 + $34,700,000 + $79,800,000 = $159,800,000. This total does not include construction of hydrologic or spillway modifications and does not include implementation and oversight costs.

7-06. Hydrologic Deficiency Remediation.

a. Recommended remediation.

There are two features which are recommended for Tuttle Creek Dam. The principal problem for this project is to provide adequate freeboard at the crest of the dam. In these circumstances, the recommended “fix” is to install short (32-inch) concrete walls in place of the upstream guardrail across the dam. This anchored “Jersey Barrier” solution has been used at Longview Lake in the Kansas City District, and at other Corps of Engineers projects. Since this solution is a low cost and effective method of addressing wave runup problems, consideration of other corrective measures such as additional spillway bays or perched auxiliary spillways were screened out because of cost.

It is noted that the short concrete barriers discussed above are adequate to deal with wave runup problems, but cannot be relied on to contain water in the lake, once the static water surface has raised above the base of the barrier. Therefore it is imperative that the existing tainter gates be structurally sound and reliable in order to keep the maximum static water level in the lake below elevation 1154.4.

b. Cost evaluation.

The detailed cost analysis is presented in Appendix VIII "Cost Estimates for the Recommended Plan" (in Volume VII of VII). The following table presents a summary of this cost analysis.

Jersey Barriers - Hydraulic Fix 05-Dec-01

Description: It is assumed wave erosion may occur. In order to prevent this erosion a barrier will be placed at the crest of the dam. It is assumed jersey barriers will be purchased and temporarily placed along the shoulder of the road for the entire 8000'. The existing guardrail will be removed and disposed. A .5' deep x 2' wide trench will be excavated and the spoil disposed offsite. 12" diameter piers will be drilled every 8' and reinforced with 2-#5bars to anchor to the pier cap. This cap will be 1.83' high x 2' wide x 8000' long. This jersey barrier will then be placed on this cap and anchored with dowels and wedge anchors. Traffic control will required. Assume 2 flagmen or 120 days each.

Item	Description	Quantity	Unit	Price/Unit	Total Price
1	Mob/Demob and Preparatory Work	1	EA	$ 77,000	$ 77,000
2	Guardrail Removal	8,000	LF	$ 3.50	$ 28,000
3	Guide Post Removal	667	EA	$ 75.00	$ 50,025
4	Trenching for Piers/Base	481	CY	$ 8.00	$ 3,848
5	Piers	1,000	EA	$ 100	$ 100,000
6	Pier Cap	1,084	CY	$ 425	$ 460,700
7	Anchors	3,200	EA	$ 27.00	$ 86,400
8	Jersey Barriers	8,000	LF	$ 27.50	$ 220,000
9	Temporarily Place Barriers	8,000	LF	$ 8.00	$ 64,000
10	Permanently Place Barriers	8,000	LF	$ 10.00	$ 80,000
11	Traffic Control	1	LS	$ 96,000	$ 96,000
		Subtotal:			$ 1,265,973
		Unlisted Items @		15%	$ 189,896
		Contingency @		25%	$ 363,967
		Cost Escalation to Oct 2001			$ 37,000
		Total Construction Cost			$ 1,856,836
				Say	$ 1,875,000

7-07. Other Deficiencies Remediation.

Tainter Gate Modifications. In order to ensure the ability to safely pass the PMF and avoid overtopping of the dam, the structural integrity of the Tainter gates must be ensured. To address this issue, detailed wind/wave and structural analyses of the Tainter gates will be performed.

The analyses will address all current guidance and all potential loading conditions. The structure of the gates will be modified and reinforced to ensure adequate strength and appropriate and safe operations during pool loading and flow conditions. These modifications are likely to include addition of bracing members, strut cover plates, modification/replacement of the trunnion pins and bearings, repositioning of the bracket for the gate dogging system and possible strengthening of the trunnion anchorage. These modifications will require repainting of the gates and associated structures and equipment. Repainting operations would involve the generation, treatment, and disposal of lead paint waste above the Resource Conservation and Recovery Act (RCRA) Land Disposal Restriction (LDR) Toxicity Characteristic Leaching Procedure (TCLP) limits. All waste generating activities will be conducted in accordance with all applicable local, state, and Federal regulations.

Given that final comprehensive gate analyses have not bee completed, a parametric evaluation of Tainter gate modifications was performed to establish a cost estimate for the Tuttle Creek Tainter gate modifications. The estimated cost per gate used for Tuttle Creek was $113,900 including 20 percent contingency. With 18 gates, the total gate modification construction cost is approximately $2.05 million. These values are considered to be conservative based on the actual contract award costs of $44,000 per gate at Garrison Dam, $110,000 per gate at Table Rock Dam, and the estimated $100,000 per gate at Harlan County Dam. Given that the gate modification estimate was prepared based on comparisons with other similar projects, a detailed estimate is not presented in Appendix VIII "Cost estimates for the Recommended Plan" (in Volume VII of VII). The costs are however included in the Total Contract Cost Summary at the end of this section.

The cost associated with tainter gate painting is presented in the following summary. The detailed cost analyses are presented in Appendix VIII "Cost estimates for the Recommended Plan" (in Volume VII of VII).

Painting of Tainter Gates, Catwalk, and Machinery 08-Mar-02

Description: The painting of tainter gates will require a portable enclosure since removal of lead-based paint will be required. An enclosure will be built that will enclose the entire gate slot both upstream and downstream. Once in place the operating machinery will be removed, blasted, and painted. The catwalk will be blasted and painted at the same time. The machinery will be reinstalled. The tainter gates will then go through the same process. Once this is done, the enclosure will be removed and placed in the next gate slot for the repeat of the process.

Item	Description	Quantity	Unit	Price/Unit	Total Price
1	Sand Blasting, and Painting of Tainter Gates	18	EA	$ 95,333	$ 1,715,994
2	Sand Blasting, and Painting of Catwalk	71,000	SF	$ 16.90	$ 1,199,900
3	Remove, Sand Blast, Paint, Reinstall Machinery	18	EA	$ 17,222	$ 309,996
		Subtotal:			$ 3,225,890
		Unlisted Items @ 0%			$ -
		Contingency @ 20%			$ 645,178
		Cost Escalation to Oct 2001			$ 79,000
		Total Construction Cost			$ 3,950,068
		SAY			$ 3,950,000

The costs associated with destructive, emergency removal of a spillway Tainter gate and replacement of the gate and damaged piers are presented below. This estimate is presented for information only and, although it is shown as Item D in Appendix VIII "Cost estimates for the Recommended Plan" (in Volume VII of VII) this item and the associated cost is not included in the Total Contract Cost Summary.

Replacement of Tainter Gates 08-Mar-02

Description: This estimate assumes, in the event of overtopping due to the gate being stuck in the closed position, the gate was removed by means of explosives. Therefore, it was assumed two piers would be damaged, and three tainter gates would have to be replaced. It is assumed the concrete around the trunion anchor pins would have to be sawcut and removed. The steel would have to be cut and new steel welded in place. The concrete would then be replaced and new tainter gates installed. It was assumed once the tainter gates are in place, they would then be lightly sandblasted and painted.

Item	Description	Quantity	Unit	Price/Unit	Total Price
1	New Tainter Gates	3	EA	$ 112,667	$ 338,001
2	Remove Damaged Concrete	2	EA	$ 7,000	$ 14,000
3	Replace Damaged Steel	2	EA	$ 20,000	$ 40,000
4	Replace Damaged Concrete	8	CY	$ 750.00	$ 6,225
5	Install Tainter Gates	3	EA	$ 25,000	$ 75,000
6	Paint Tainter Gates	3	EA	$ 59,000	$ 177,000
		Subtotal:			$ 650,226
		Unlisted Items @		10%	$ 65,023
		Contingency @		100%	$ 715,249
		Cost Escalation to Oct 2001			$ 29,000
		Total Construction Cost			$ 1,459,497
				SAY	$ 1,460,000

It was specified to use some of the riprap of the upper portion of the dam to build the upstream toe of the working platform on the upstream slope. The costs associated with restoration of the original upstream shell of the dam are summarized in the following table:

Upstream Riprap Overlay 08-Mar-02

Description: Once the upstream remediation is complete the exposed face will be resurfaced with a riprap overlay. Large size rock (33"-2800lb) stone will be used for the rockfill area from which the dike structure material was taken, and a smaller size rock (24" - 1100lb) stone will be used for the face of the working platform. It is assumed this rock is coming from a local quarry within 18 miles from the job site. For the top of the working platform, 6" bedding material will be used. This material will come from a quarry approximately 9 miles away. Rock will be placed using a crane, hydraulic excavator, and frontend loader.

Item	Description	Quantity	Unit	Price/Unit	Total Price
1	Rockfill Replacement	115,741	CY	$ 47.18	$ 5,460,660
2	Working Platform Face Protection	41,667	CY	$ 37.97	$ 1,582,096
3	Working Platform Horizontal Layer	21,296	CY	$ 23.71	$ 504,928
		Subtotal:			$ 7,547,685
		Unlisted Items @		15%	$ 1,132,153
		Contingency @		20%	$ 1,735,967
		Cost Escalation to Oct 2001			$ 214,000
		Total Construction Cost			$ 10,629,805
				SAY	$ 10,630,000